STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION

SOIL MECHANICS BUREAU

FOUNDATION INVESTIGATION REPORT

NEW YORK STATE THRUWAY INTERCHANGE
WITH ROUTE 9W AT COXSACKIE
GREENE COUNTY
PROJECT NUMBER 1416.00

SM 238a (1/69)

FOUNDATION INVESTIGATION REPORT

April 25, 1969

TO: Vernon J. Burns
Deputy Chief Engineer (Design)

FROM: Wm. P. Hofmann, Director
Bureau of Soil Mechanics

PROJECT: New York State Thruway
Interchange with Route 9W
at Coxsackie
Greene County
Project No. 1416

We have completed a foundation investigation and are hereby transmitting a Foundation Investigation Report for the above project. This study has been prepared and is presented in detail to allow the Department of Transportation to make an extensive review of the foundations design to be proposed by the Contracting Engineers for this project, Madigan-Hyland, Inc. In addition, it will, in our opinion, form a basis for the discussion and resolution of questions on this subject which may arise.

SUMMARY

Our investigation indicates that the approach embankments can be safely constructed without special foundation treatment. Structure abutments should be supported on end-bearing piles. The piers may be supported on spread footings placed on 2 feet of compacted Item 2 VJ-D and designed for a maximum allowable bearing pressure of 2 tsf. Structure settlements will be negligible at the abutments and about 1.5 inches at the piers. Differential settlement of the approach embankments with respect to the abutments will necessitate some post-construction maintenance. Drag forces on the piles have been computed and are quite significant. Also, the use of lightweight fill as embankment material in the abutment areas would reduce post-construction settlement, but at a high cost.

Detailed recommendations for foundation design on the above project and a description of our investigations and analyses are given in the body of this report.

We are enclosing three extra copies of this report for transmittal to the District or others.

Bernard E. Butler

Bernard E. Butler
Associate Soils Engineer

TJN/PJW:B
Enc.
cc: G. W. McAlpin

FOUNDATION INVESTIGATION REPORT

New York State Thruway Interchange

with Route 9W at Coxsackie

Greene County

Project No. 1416.00

New York State Department of Transportation

Bureau of Soil Mechanics

April 1969

I. INTRODUCTION

A. Subject and Scope of Report

This project consists of the construction of a new Thruway Interchange in Greene County, approximately 3 miles north of Coxsackie. A toll plaza will be located between Route 9W and the Thruway. It is assumed that a culvert will carry a small stream under the toll plaza embankment, the maximum height of which will be 21 feet. Traffic using the northbound Thruway lanes will cross the Thruway on a structure having two simply supported spans, each approximately 100 feet long. The approach embankments to the structure will have a maximum height of 25 feet above the surrounding ground surface.

Our investigation consisted of the following items of work:

1. Analysis of the stability of the structure approach embankments.
2. Settlement analysis for the structure and the approach embankments.
3. Settlement analysis in the toll plaza area.
4. Analysis for the foundation design recommendations for the structure.

B. Basis of Report

Our investigation was based on the following information:

1. A one inch equals 20 feet preliminary plan and profile

of the structure, prepared by Madigan-Hyland, Inc., the Contracting Engineers for this project.

2. A one inch equals 200 feet plan and a one inch equals 50 feet horizontal by 10 feet vertical profile for the interchange, prepared by Madigan-Hyland, Inc.

3. Interpretation of borings logs and visual examination and classification tests on soil samples from one 4 inch diameter cased undisturbed sample drill hole, numbered UDH-4, and from twelve 2 1/2 inch diameter cased drill holes numbered DH-1 to DH-3, DH-5 to DH-9, DH-10, DH-11 and DH-18. In UDH-4, undisturbed 3.375 inch diameter samples were obtained, generally at 10 foot intervals. The borings were made by Contract under the supervision of Madigan-Hyland, Inc.

4. Results of laboratory tests performed by this Bureau on undisturbed samples from UDH-4.

5. A field investigation by representatives of this Bureau of the existing Thruway bridge over the Penn Central Railroad, located 0.8 miles north of the proposed Interchange in similar soil conditions.

6. Logs of borings and results of laboratory tests performed during investigations made for the original design of the Thruway.

II. SITE CONDITIONS

A. Topography and Geology

The project is situated on a terrace of the Hudson River Valley at an elevation about 150 feet above sea level. The topography of the site is one of generally flat land broken by occasional stream-carved depressions. A small stream runs through the site of the proposed toll plaza. Route 9W is located along the foot of "High Rocks" at the western boundary of the terrace, about 1,000 feet west of the Thruway. The Hudson River lies about 1 1/2 miles east.

The bedrock geology in this area is extremely complex. Bedrock consists of folded and faulted Ordovician sandstones and shales. Bedrock expressions on the surface are in the form of ridges which have their axes aligned generally in a north-south direction. A thin layer of glacial till is found over the bedrock.

At the end of the most recent glaciation, this area was a part of glacial Lake Albany, which extended as far south as Kingston. During this period, meltwater from the continental glacier deposited thin, alternating layers, or varves, of silts and clays on the lake bottom.

The main inflow of water into the lake took place north of the project area. As a result, the clays around Albany are composed of coarser particles that settled out before reaching

the Coxsackie area. This can be noted in the somewhat higher moisture contents and plasticity indices observed in this investigation as compared to the typical Albany clays.

Eventually, as a result of uplift and/or other causes, the lake waters escaped down the Hudson River Valley to the ocean. As the lake drained, the sediments were exposed to surface drying. In subsequent periods of climate change accompanied by low rainfall, the ground water level subsided resulting in increased consolidation, or precompression, of the varved silts and clays.

B. Subsurface Conditions

The subsurface conditions are shown in Drawing 1 SM 1960A contained in the Appendix to this report. The soil profile is composed of the following strata:

1. A surface layer of about 25 feet of stiff to hard, desiccated, brown, layered silt and clay. This material has a comparatively high shear strength and low compressibility.

2. About 35 to 70 feet of firm, sensitive, grey, layered silt and clay. This material has a limited shear strength and is highly compressible at pressures greater than its maximum past consolidation pressure.

3. About 5 feet of very dense glacial till consisting of mixtures of sand, silt and gravel with a trace of

clay and possibly containing boulders, having high strength and low compressibility.

4. A sloping bedrock surface encountered at a maximum depth of 100 feet in UDH-4.

Due to the low permeability of these soils and the time lag included, it is impossible to state, based on the short-term observations made in the borings, at what depth the static groundwater table occurs. In DH-5, DH-7 and DH-8, artesian water, originating in the till or the bedrock, flowed out of the casing. No attempt was made to measure the artesian head. An artesian head was measured at a location 0.5 miles from the proposed interchange in borings made for the original design of the Thruway. The measurements indicated that the artesian head was approximately 10 feet above ground surface.

III. LABORATORY TESTING PROGRAM

A. Summary and Discussion of Testing

The following types and number of tests were performed:

1. A moisture content determination was made on every sample of cohesive soil received in the laboratory.

2. Liquid limit, plastic limit, and grain-size (hydrometer) tests were performed on undisturbed samples from boring UDH-4.

3. The following tests were performed on undisturbed samples from boring UDH-4:

a. Seven consolidation tests were run in the usual manner except that the load was recycled in order to ascertain the recompression characteristics of the soil. The compression index and estimated preconsolidation pressure obtained from each consolidation test are plotted in Drawing No. 1 SM 1960B, included in the Appendix. Other properties used in our analyses and based on consolidation test results are also shown in this drawing. The rate at which consolidation or settlement will occur, as interpreted from laboratory test results, is greatly affected by sample disturbance. This factor and other inadequacies of analysis procedures for time rate of settlement requires that considerable judgment be used.

b. Seven unconsolidated-undrained (UU) triaxial shear strength tests were performed at confining pressures equal to the estimated total overburden pressure in the ground at the depth at which the sample was obtained.

c. Three sets of three consolidated-undrained (CU) triaxial shear strength tests were performed. Each set of three samples was from the same depth in the boring. Individual samples were consolidated

prior to failure at the estimated effective overburden pressure and two higher pressures, respectively, in order to obtain an indication of the increase in shear strength with consolidation pressure.

d. One set of three consolidated-drained (CD) triaxial shear strength tests, each consolidated prior to failure at different confining pressures. The samples were sheared at a slow rate of deflection (0.000096 inches per minute). The resulting effective stress strength parameters were found to be comparable to those typical of Albany clays.

e. Eight "undisturbed" and eight remolded laboratory vane shear strength tests were performed. Based on a comparison of the remolded shear strengths with the original shear strengths measured, the upper brown silt and clay layer can be classified as relatively insensitive to disturbance while the lower grey silt and clay is sensitive to disturbance.

Shear strengths from all tests excepting the consolidated-drained (CD) type are plotted on Drawing No. 1 SM 1960B in the Appendix.

IV. DESIGN ANALYSES

A. Selection of Shear Strength Parameters

The upper layer of brown silt and clay has been preconsolidated by desiccation. A shear strength of 1600 psf was selected for use in stability analyses based on shear strength results shown on Drawing No. 1 SM 1960B in the Appendix.

The underlying gray silt and clay has a lower strength. Drawing No. 1 SM 1960B contains shear strength results determined from the various types of tests, including those from CU tests for three artesian pressure assumptions. The artesian head in each case was assumed to dissipate its bouyant or uplift effect uniformly from the full value at a depth of 90 feet to zero at a depth of 10 feet; i.e., the assumed groundwater level. Also shown in the shear strength plot of this drawing is the theoretical variation of shear strength with depth (assuming no precompression and no artesian head) based on a shear strength to consolidation pressure (c/p) ratio of 0.25. This value of the c/p ratio has been found to be generally valid for glacial lake clays of New York State. After giving due consideration to: 1) the decrease in strength in UU tests as a result of sampling disturbance; 2) the strengths obtained from CU tests, including the effects of various assumptions regarding the groundwater level and the possible artesian pressures, as well

as the effects of disturbance and reconsolidation; and 3) the preconsolidation evident from consolidation tests, a uniform shear strength of 700 psf in the gray silt and clay was selected for design purposes. This value is considered to be on the conservative side.

B. Embankment Stability

The stability against a rotational failure for the 22 feet high end fill and 25 feet side fill conditions in the structure area was analyzed using a computer program normally employed by this Bureau for this purpose . The program automatically searches for the failure arc having the lowest safety factor. The minimum factor of safety was found to be 1.45 for the side fill and 1.51 for the end fill conditions at the west approach embankment. It is estimated that the safety factor against a rotational shear failure at the toll plaza and the east approach embankment is higher because of the smaller thickness of the grey clay layer. The results of these analyses for the circles giving the lowest safety factors are shown in Drawing No. 1 SM 1960C, included in the Appendix to this report.

C. Embankment Settlement

Settlement of the embankments depends, among other variables, on the existing effective stress in the soil and on the maximum past pressure under which the soil has been consolidated (the preconsolidation pressure). The interpretation of the precon-

solidation pressure from consolidation tests is frequently made difficult by disturbance during sampling and preparation of samples for testing. The stress condition in the soil is not accurately known because of the indefinite location of the groundwater level and the unknown magnitude of artesian pressure. In the settlement analysis for this project conservative assumptions were made to account for these factors and the resulting estimates are believed to be the maximums anticipated for this project.

In computing the rate of settlement, assumptions were made regarding the drainage characteristics of the compresible soils, including the effect of lateral drainage.

The approach embankment settlement can be reduced by the use of lightweight fill (dry unit weight equal to 70 lbs. per cubic foot). It is estimated that the cost of lightweight fill at this location would be $8 per cubic yard in place. A suggested specification for lightweight fill can be submitted, if desired.

The anticipated centerline settlements of the two approach embankments and the toll plaza at the assumed drainage structure location 1 and 5 years after the end of construction, and ultimately, are given in the following tabulation. The settlements are higher at the west approach because of the greater

depth of the silt and clay layer at that location. It should be noted that the settlements will also be the differential settlements between approach embankments and pile-supported abutments.

Location	Total Settlement After		Ultimate Settlement
	1 year	5 years	
East Approach Embankment	5.5 in.	6.5 in.	7 in.
West Approach Embankment	8 in.	11.5 in.	15 in.
West Approach Embankment Using Lightweight Fill	5 in.	6 in.	7 in.
Toll Plaza (assumed culvert location)	4 in.	4.5 in.	5 in.

The above tabulation shows that by the use of lightweight fill the settlement of the west approach embankment can be reduced to that expected at the east approach embankment.

D. Structure Foundation

1. Abutments

If the abutments were supported on spread footings, the abutment settlements would be similar to the expected approach embankment settlements listed above. Therefore, to reduce structure settlement, the abutments should be supported on end bearing piles. As a result of approach embankment settlement, the abutment piles will be subject

to pile drag. We estimate that regular-weight fill will create drag forces which may reach as much as 100 tons per vertical pile at the west abutment and 60 tons per vertical pile at the east abutment. Utilizing lightweight fill in the west approach embankment, the estimated drag would be reduced to about 70 tons per vertical pile. Drag forces on piles battered away from the center of the approach embankments are anticipated to be about one-half of the magnitude of those on vertical piles.

Although these drag estimates may appear to be inordinately high, the poor performance of the nearby Thruway bridge on piles over the Penn Central Railroad tends to substantiate the potential drag problem in this area. Cracks were noted in the abutments of this existing Thruway bridge, which is only about 0.8 miles north of the project site. These cracks are similar to those observed at bridges in New York State where structure settlement, as a result of pile drag, has been measured.

Cracking of abutments is more common in cases where pile-supported structures have experienced pile drag than in the case of abutments on spread footings, even though the latter may have undergone considerably greater settlement. This is because the soil under a spread footing foundation will adjust to provide a reasonably

uniform settlement under this type of foundation, whereas some piles in a long pile group will be non-uniformly over-loaded and will settle more than others. This situation results in the removal of continuous support from beneath the pile cap.

The effects of pile drag could be reduced by supporting the abutments on heavy steel H-piles driven to practical refusal. It would be desireable that the H-piles be provided with reinforced driving points in order to ensure that they penetrate to bedrock. In our opinion, the allowable stress in the H-piles may be increased if pile drag is included in the design pile loads.

2. Pier

The dead and live load for the pier is small compared to the load imparted by the approach embankments in the abutment areas. Our analysis indicates that the pier may be supported on spread footings designed for a maximum bearing pressure of 2 tons per square foot without increasing the pressure at any depth in the foundation soils above the preconsolidation pressure. The anticipated ultimate pier settlement for a uniform bearing pressure of 2 tons per square foot is 1.5 inches, all of which will occur within 6 months after construction.

The brown layered silt and clay at footing level is

subject to softening from disturbance under construction traffic possibly combined with exposure to rain. Therefore, before construction of the pier footing, the underlying foundation soils should be excavated to a depth of two feet below footing base elevation and replaced with compacted Item 2VJ-D.

V. Conclusions and Recommendations

1. The embankments can be safely constructed to the proposed heights using a normal construction procedure.

2. Using regular fill and a normal construction procedure, the estimated settlement of the west abutment is 15 inches, while that of the east abutment is 7 inches.

3. The west approach embankment may be partially constructed of lightweight fill to reduce settlement of the embankment and drag on piles supporting the abutment. The upper limit of lightweight fill should be 2 feet below subgrade elevation. The lateral limits at the front and the sides of the approach embankment should be the normal embankment side slopes. The lateral limit in back of the abutment should be a plane intersecting the upper limit 20 feet back of the wingwalls and inclined at a slope of 1 vertical on 2 horizontal away from the abutment. The cost of using lightweight fill should be

compared to the cost of additional maintenance and, possibly, an initial temporary pavement to be replaced by a permanent one after most of the settlement has taken place.

4. The anticipated west approach embankment settlement using lightweight fill is 7 inches. Most of this settlement will occur within 1 year after the end of construction.

5. The requirements of this project do not appear to permit any waiting periods. However, in order to reduce the differential settlement between the approach embankments and the abutments, embankment construction and paving operations should be scheduled so as to provide the maximum possible time interval between completion of the embankments to subgrade elevation and paving.

6. We recommend that settlement platforms be included in the contract in order to verify the design assumption and the predicted settlements. At least two settlement platforms should be installed at ground surface under each approach embankment near the abutment areas.

7. Abutments should be supported on heavy steel H-pile sections with reinforced driving tips. Piles should be driven to practical refusal. Abutment settlement will be less than one inch.

8. Pile drag of considerable magnitude is anticipated. It is our opinion that the allowable stress in the pile may be increased if drag loads are included in the design pile loads.

9. The pier may be supported on a spread footing foundation designed for a maxium bearing pressure of 2 tons per square foot and placed on a 2 feet thick layer of compacted Item 2VJ-D.

10. The anticipated settlement of the pier is 1.5 inches, all of which will occur during the first 6 months after construction. Placing the pier on spread footings would have the added advantages of not having to move the pile driving rig across the Thruway and of not having the pile driving rig in the median, thereby creating a traffic hazard.

11. The maximum settlement at the proposed toll plaza will be about 5 inches which will occur above the existing stream channel. This settlement should be taken into account when designing a culvert under the toll plaza.

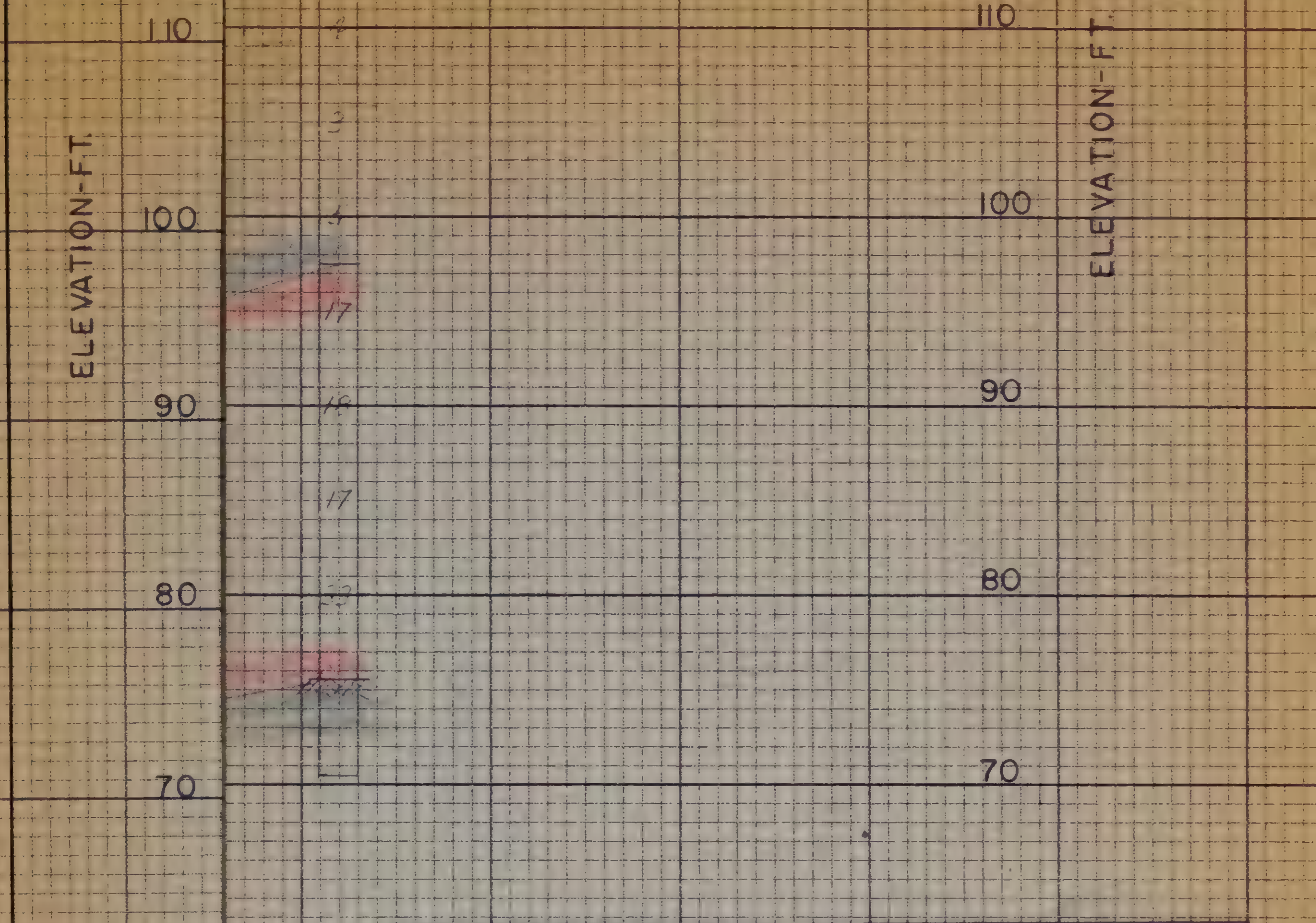
We will be pleased to review and discuss this report in further detail and be of further service on this project, if you so desire.

Bernard E. Butler

Bernard E. Butler
Associate Soils Engineer

TJN/PJW/ARS:B

APPENDIX



PREPARED BY:

DRAWN BY:

CHECKED BY:

STATE OF NEW YORK

DEPARTMENT OF TRANSPORTATION

BUREAU OF SOIL MECHANICS

NEW YORK STATE THRUWAY INTERCHANGE

WITH ROUTE 9W AT COXSACKIE

PROJECT NO. 1416.00

SUBSURFACE PROFILE

RAMP A & B STATION 997 TO STATION 1007

BOR

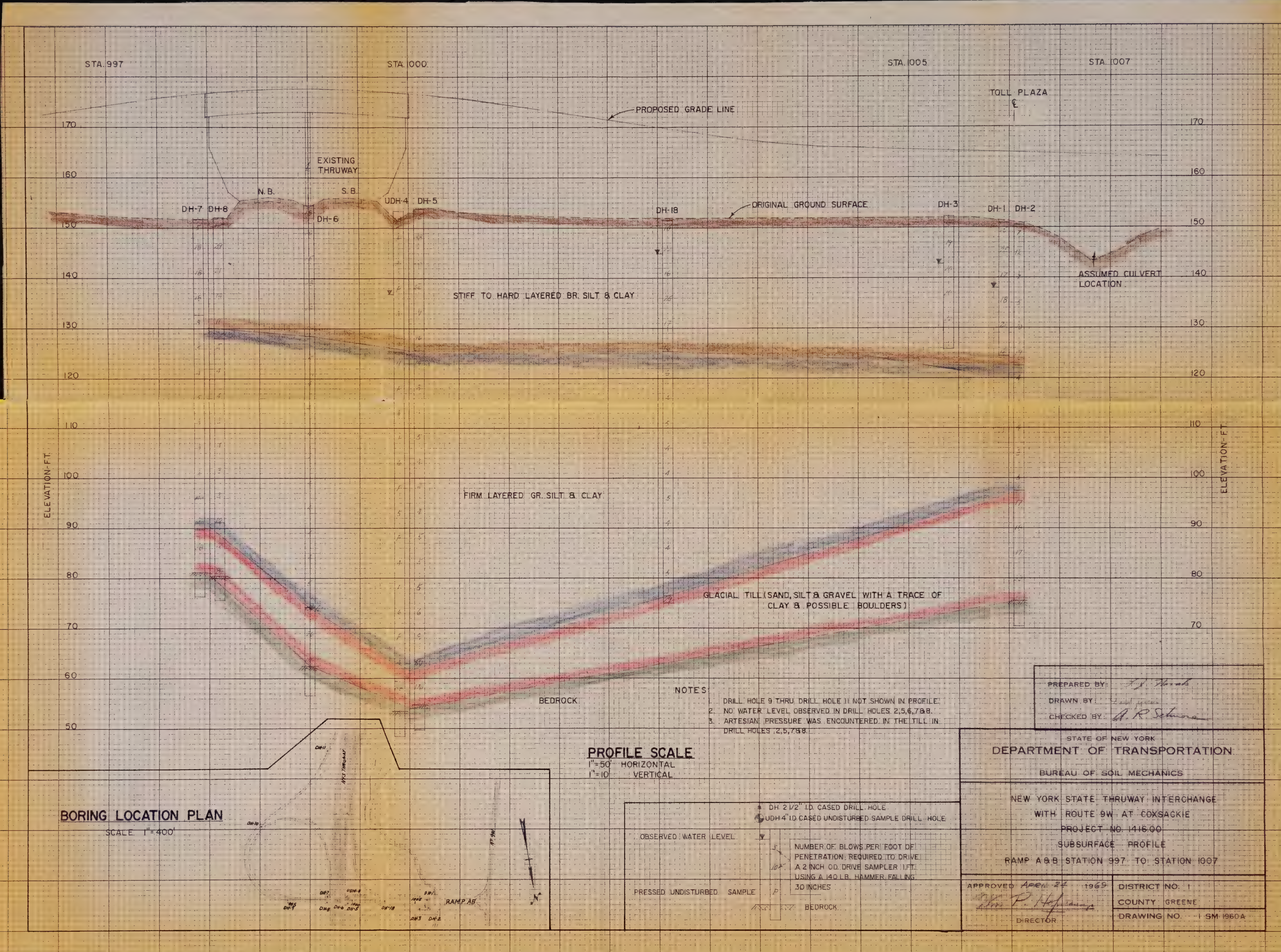
OVED April 24 1969

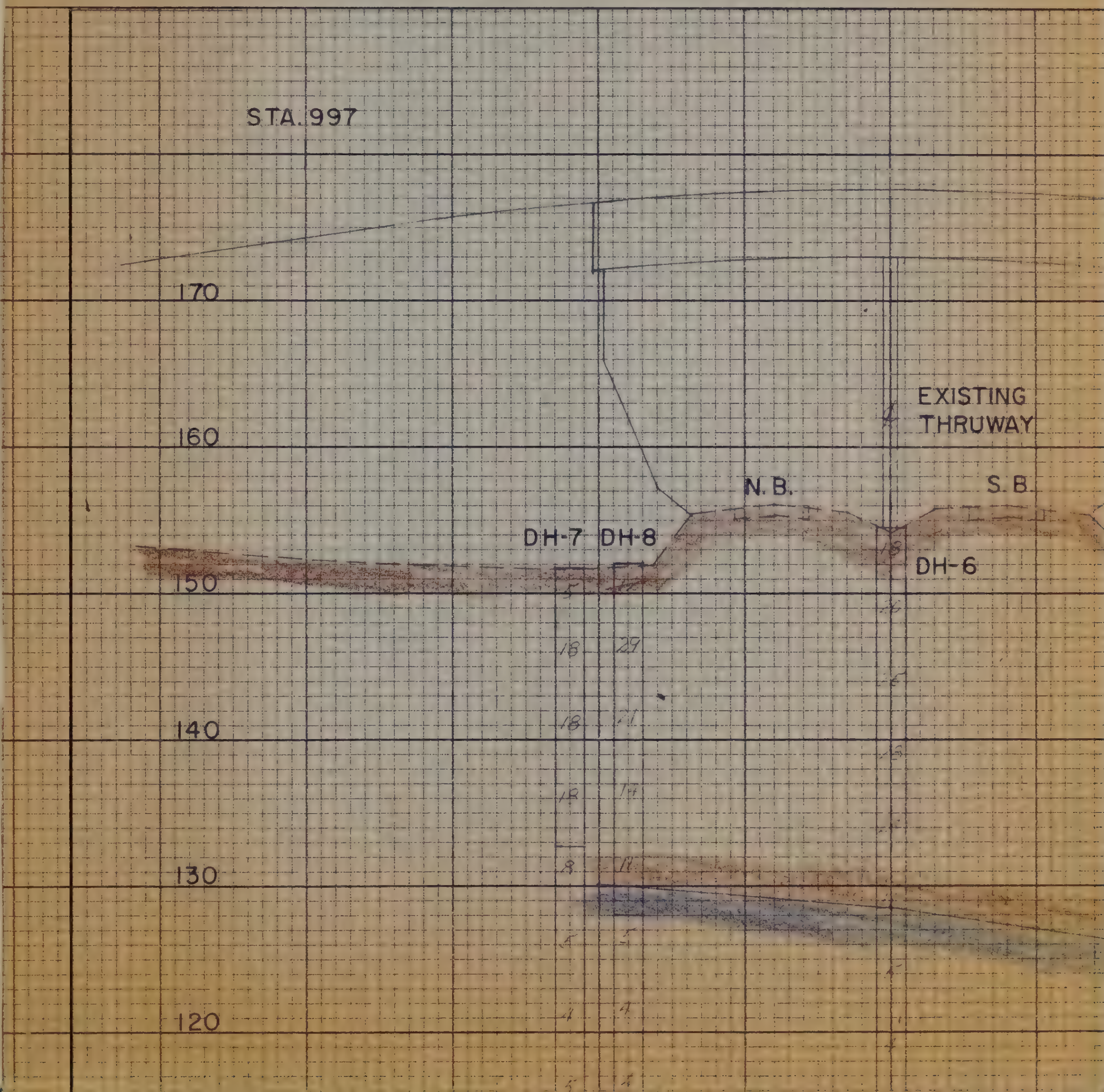
DIRECTOR

DISTRICT NO. 1

COUNTY GREENE

DRAWING NO. 1 SM 1960A


STA. 997
STA. 1000
STA. 1005
STA. 1007
TOLL PLAZA
PROPOSED GRADE LINE
EXISTING THRUWAY
N.B.
S.B.
DH-7
DH-8
DH-6
UDH-4
DH-5
DH-18
ORIGINAL GROUND SURFACE
DH-3
DH-1
DH-2
ASSUMED CULVERT LOCATION
STIFF TO HARD LAYERED BR. SILT & CLAY
FIRM LAYERED GR. SILT & CLAY
GLACIAL TILL (SAND, SILT & GRAVEL WITH A TRACE OF CLAY & POSSIBLE BOULDERS)
BEDROCK
ELEVATION-FT.
170
160
150
140
130
120
110
100
90
80
70
60
50
NOTES:
1. DRILL HOLE 9 THRU DRILL HOLE 11 NOT SHOWN IN PROFILE.
2. NO WATER LEVEL OBSERVED IN DRILL HOLES 2,5,6,7 & 8.
3. ARTESIAN PRESSURE WAS ENCOUNTERED IN THE TILL IN DRILL HOLES 2,5,7 & 8.
PROFILE SCALE
1"=50' HORIZONTAL
1"=10' VERTICAL
BORING LOCATION PLAN
SCALE 1"=400'
RAMP AB
DH 2 1/2" I.D. CASED DRILL HOLE
UDH 4" I.D. CASED UNDISTURBED SAMPLE DRILL HOLE
OBSERVED WATER LEVEL
NUMBER OF BLOWS PER FOOT OF PENETRATION REQUIRED TO DRIVE A 2 INCH O.D. DRIVE SAMPLER 1 FT. USING A 140 LB. HAMMER FALLING 30 INCHES
PRESSED UNDISTURBED SAMPLE
BEDROCK
PREPARED BY:
DRAWN BY:
CHECKED BY: A. R. Selmer
STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
BUREAU OF SOIL MECHANICS
NEW YORK STATE THRUWAY INTERCHANGE WITH ROUTE 9W AT COXSACKIE
PROJECT NO. 1416.00
SUBSURFACE PROFILE
RAMP A & B STATION 997 TO STATION 1007
APPROVED April 24 1969
DIRECTOR
DISTRICT NO. 1
COUNTY GREENE
DRAWING NO. 1 SM 1960A



STA. 997
170
160
150
140
130
120
EXISTING
THRUWAY
N. B.
S. B.
DH-7
DH-8
DH-6

SM

ASSUM
G.W.L.

UNDISTURBED DRILL HOLE NO. 4

LABORATORY PROJECT NO. 1-447

STATION 20+56 OFFSET 9.5' RT.

SURFACE ELEVATION 153.4

OBSERVED GROUND WATER ELEVATION 136.7'

DESIGN VALUES

C_c = 0.50
C_r = 0.035
C_v = 0.10 FT^2/ DAY

DEPTH (ft.)

PREPARED BY: Ralph Wilson
DRAWN BY: F. Agostino
CHECKED BY: Gustav R. Schnore

STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
BUREAU OF SOIL MECHANICS

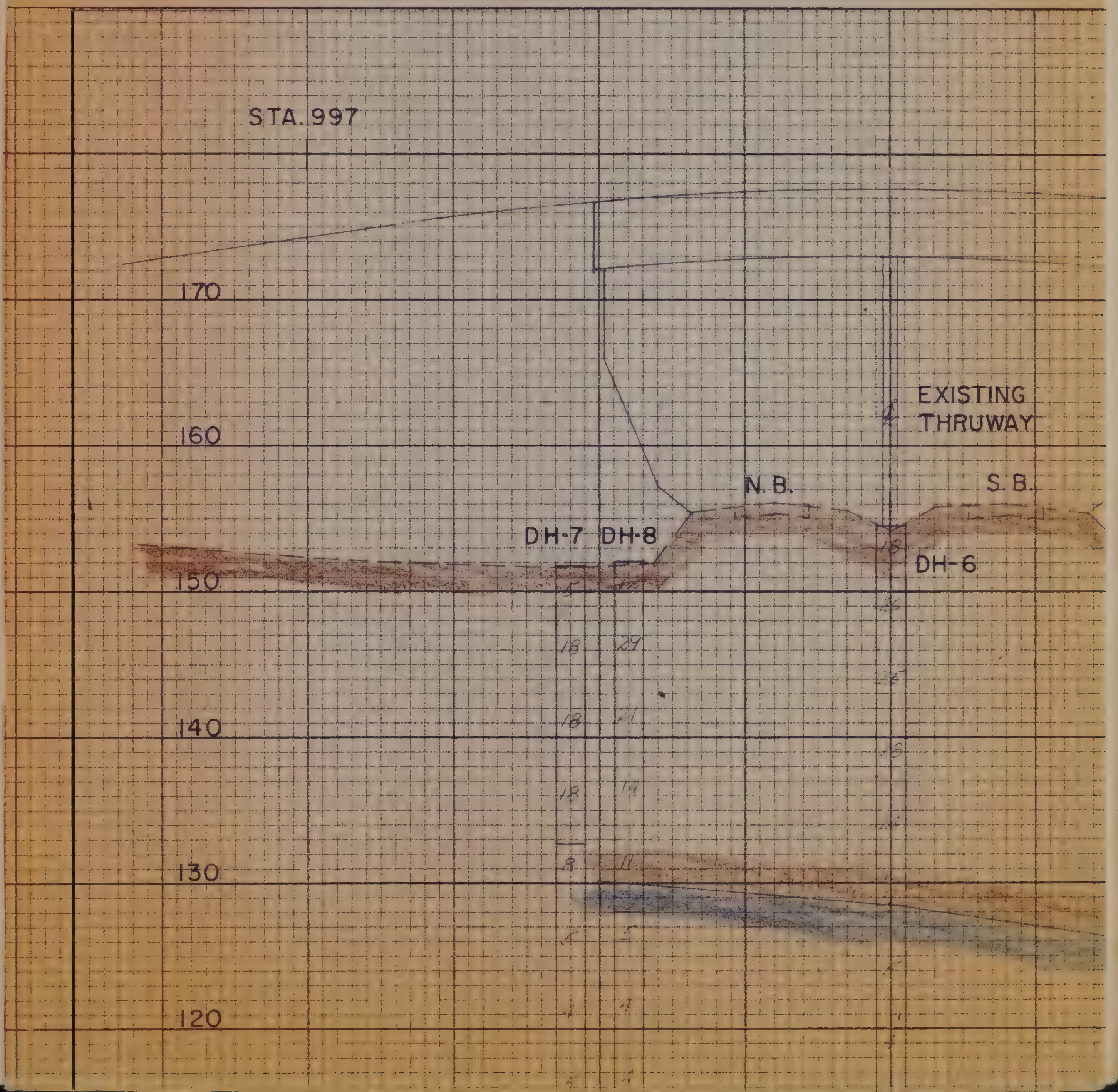
SUMMARY OF LABORATORY TEST RESULTS

N.Y.S. THRUWAY INTERCHANGE
WITH RTE. 9W AT COXSACKIE
PROJECT NO. 1461.00

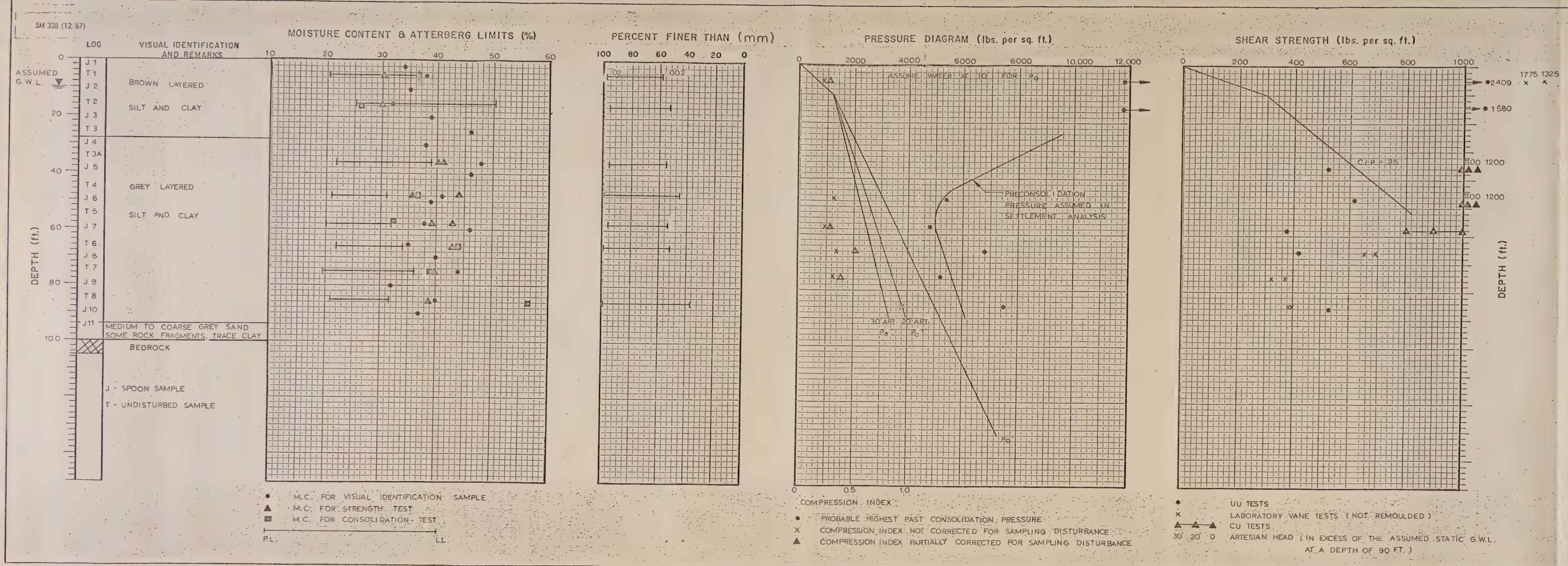
APPROVED APRIL 23 1969
Wm. P. Hofmann
PRINCIPAL SOILS ENGINEER

DISTRICT NO. 1
COUNTY GREENE

DRAWING NO. 1 SM 1960B



STA. 997
170
160
150
140
130
120
EXISTING
THRUWAY
N. B.
S. B.
DH-7
DH-8
DH-6



Sheet 1 of 1

UNDISTURBED DRILL HOLE NO. 4

LABORATORY PROJECT NO. 1 - 447

STATION 20 + 56 OFFSET 9.5' RT.

SURFACE ELEVATION 153.4

OBSERVED GROUND WATER ELEVATION 136.7'

DESIGN VALUES

C_c = 0.50
C_r = 0.035
C_v = 0.10 FT^2/ DAY

PREPARED BY:

DRAWN BY:

CHECKED BY:

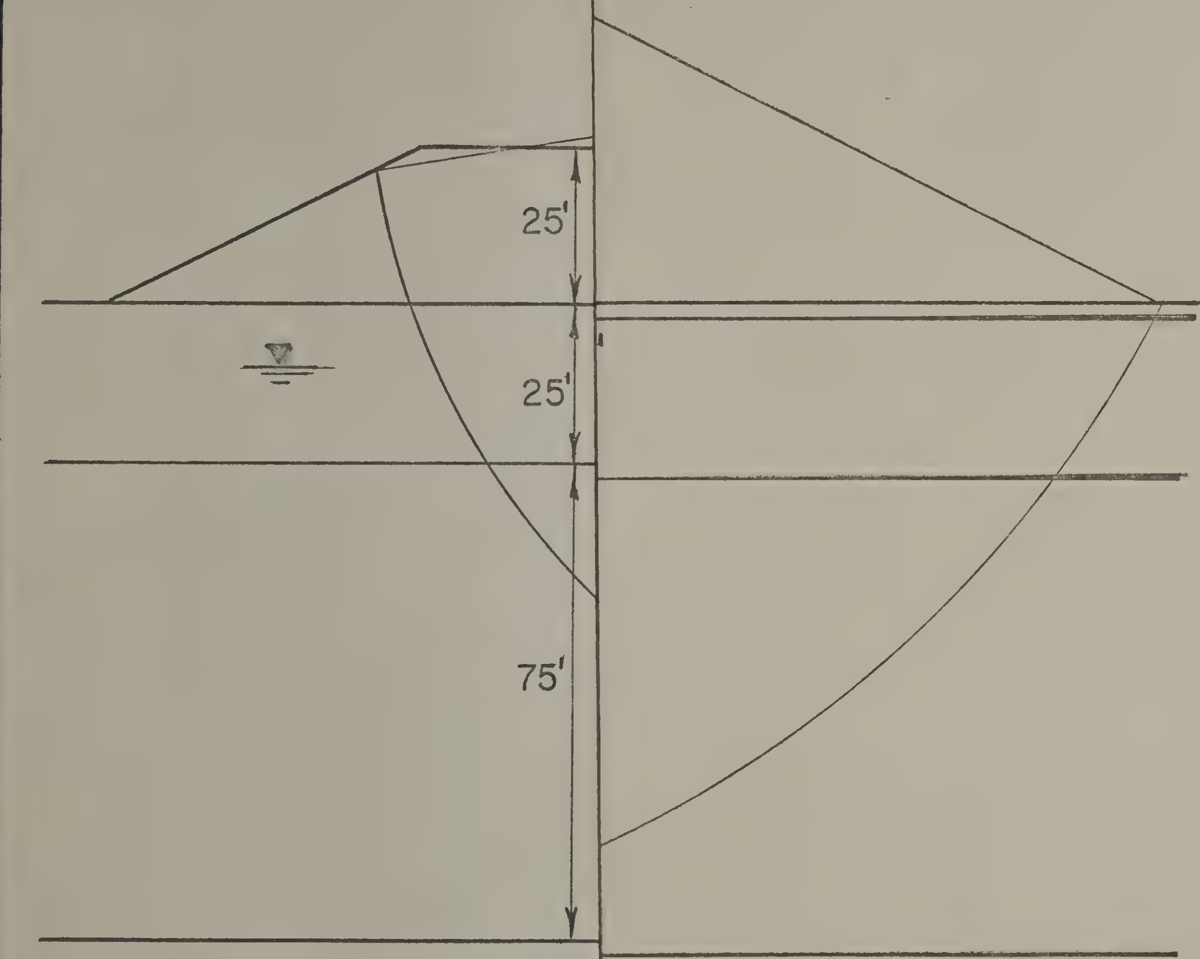
STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
BUREAU OF SOIL MECHANICS

SUMMARY OF LABORATORY TEST RESULTS

N.Y.S. THRUWAY INTERCHANGE
WITH RTE. 9W AT COXSACKIE
PROJECT NO. 1461.00

APPROVED APRIL 23 1969
PRINCIPAL SOILS ENGINEER

DISTRICT NO. 1
COUNTY GREENE
DRAWING NO. 1 SM 1960B



STATE OF NEW YORK

PARTMENT OF TRANSPORTATION

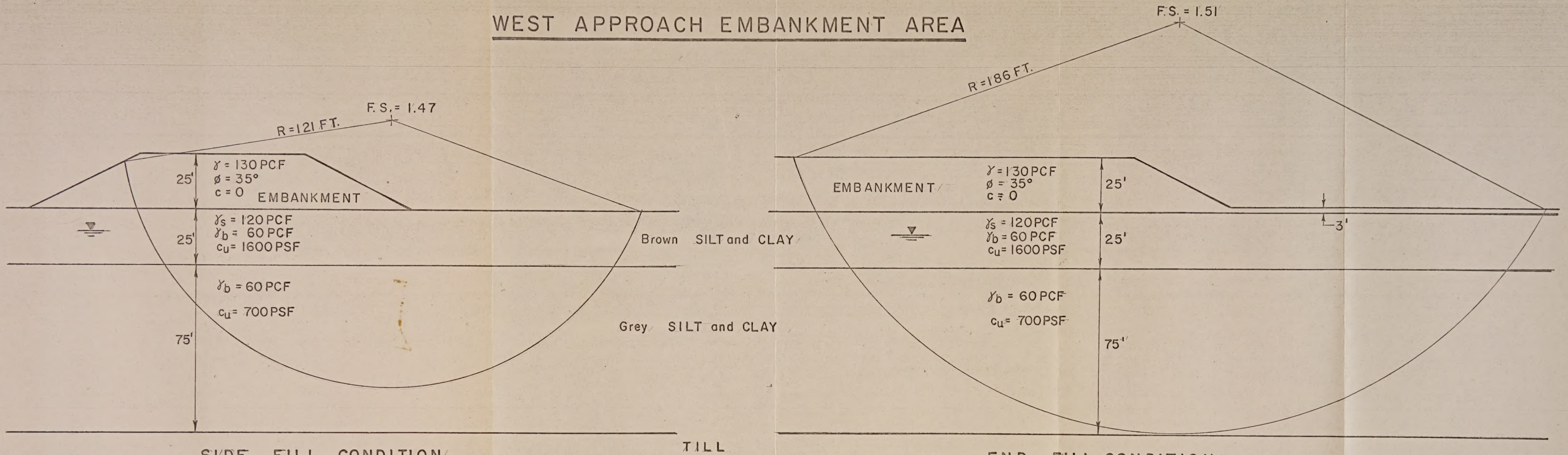
BUREAU OF SOIL MECHANICS

YORK STATE THRUWAY INTERCHANGE
VITH ROUTE 9W AT COXSACKIE
PROJECT NO. 1416.00

MMARY OF STABILITY ANALYSES

19	DISTRICT NO 1
	COUNTY GREENE
DIRECTOR	DRAWING NO. 1 SM 1960C

WEST APPROACH EMBANKMENT AREA



SIDE FILL CONDITION

END FILL CONDITION

EXPLANATION OF SYMBOLS

γ = Unit Weight
γ_s = Unit Weight (Saturated)
γ_b = Unit Weight (Bouyant)
ϕ = Angle of Internal Friction
c_u = Shear Strength used in ϕ_u = 0 Analysis
F.S. = Factor of Safety

PREPARED BY: A. R. Schnore
DRAWN BY: J. J. Mori
CHECKED BY: B. E. Butler

STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
BUREAU OF SOIL MECHANICS

NEW YORK STATE THRUWAY INTERCHANGE
WITH ROUTE 9W AT COXSACKIE
PROJECT NO. 1416.00

SUMMARY OF STABILITY ANALYSES

APPROVED April 24 1969
Wm. P. Hofmann
DIRECTOR

DISTRICT NO 1
COUNTY GREENE
DRAWING NO. 1 SM 1960C